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## CIVIL & ENVIRONMENTAL ENGINEERING | RESEARCH ARTICLE

# Simplified empirical approach for predicting the remaining strength factor used in pavement rehabilitation applications

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**Abstract:** This paper presents a simplified empirical model for predicting the asphaltic remaining strength factor to be used in estimating the resurfacing thickness for both thin and thick asphaltic surfaces. The proposed model for predicting the asphaltic remaining strength factor in the case of thin asphaltic surface is mainly a function of key performance indicators and calibration constant (K). In the case of thick asphaltic surface, an average remaining strength factor is proposed which is a function of the existing asphaltic surface thickness, cold milling thickness, and the remaining strength factor associated with thin asphaltic surface. The proposed remaining strength factor is to be used in estimating the resurfacing thickness component due to the strength loss endured by the asphaltic surface. Two case studies are presented to predict the remaining strength factor. The first one applies the remaining strength factor model to estimate the resurfacing thicknesses for two sample projects considering variable rehabilitation scheduling time, while the second one calibrates the remaining strength factor model for a local roadway sample using minimization of the sum of squared errors. The sample results

## ABOUT THE AUTHOR



Khaled A. Abaza

Prof. Abaza earned his PhD in Transportation Engineering Systems from the University of Toledo, Ohio, 1990. He is a licensed professional engineer (PE) in the State of California where he worked as a transportation engineer with the California Department of Transportation (Caltrans) from 1991 to 1997. He joined Birzeit University in 1997 and held different teaching and administrative positions including chairperson of the Civil Engineering Department and dean of the Faculty of Engineering and Technology. Prof. Abaza has mainly applied his expertise in stochastic modeling, optimization methods, statistical analysis and pavement engineering to provide reliable solutions to many of the contemporary problems facing pavement engineers. His recent research efforts have mainly focused on pavement performance prediction using discrete-time heterogeneous Markov chains. This has led to the development of Empirical-Markovian models which are used in developing optimal pavement rehabilitation strategies and predicting resurfacing design thicknesses.

## PUBLIC INTEREST STATEMENT

Pavement rehabilitation and management plays a key role in maintaining the road network at an acceptable quality level. Major rehabilitation strategies of asphaltic pavement typically include plain resurfacing, cold milling and resurfacing, and reconstruction. Former studies have indicated that plain resurfacing, and cold milling and resurfacing are the most feasible alternatives provided their timing schedules are optimally determined. Determination of relevant layers' thicknesses largely depends on the estimation of the remaining strength of the existing asphaltic surface layer. This paper mainly proposes a simple empirical approach to estimate the remaining strength in terms of "remaining strength factor" to be used in estimating the layers' thicknesses associated with both plain resurfacing (plain overlay), and cold milling and resurfacing. The proposed remaining strength factor is mainly a function of key pavement performance indicators such as the international roughness index (IRI) and present serviceability index (PSI) which are readily available to highway agencies.

indicate that the remaining strength factor values (0.45–0.94) are lower for thin asphaltic surface compared to the corresponding values (0.72–0.97) for thick surface considering 6–12 years rehabilitation scheduling time, and they are lower for inferior pavement performance compared to a superior one. The sample results also indicate that the optimal (K) values for thin asphaltic surface (0.71–1.24) are considerably lower than the corresponding optimal (K) values for thick surface (2.08–3.83).

**Subjects:** Engineering & Technology; Transportation Engineering; Pavement Engineering

**Keywords:** flexible pavement; overlay design; pavement performance; pavement rehabilitation; pavement management

### 1. Introduction

Prediction of pavement remaining strength is vital to pavement management applications as it allows pavement engineers to decide on the potential rehabilitation strategies and specify appropriate resurfacing thicknesses. The proposed remaining strength factor provides a means to estimate the remaining strength associated with asphaltic surfaces so that a pavement structure can be redesigned for the strength loss it has endured over its service life. Two general approaches are currently practiced in estimating the pavement remaining strength. The first approach is a mechanistic-empirical one that mostly relies on surface deflection measurements obtained using non-destructive testing procedures such as the falling-weight deflectometer (FWD) procedure (Hoffman, 2003; Nam, An, Kim, Murphy, & Zhang, 2016; Sarker, Mishra, Tutumluer, & Lackey, 2015; Smith et al., 2017; Tutumluer & Sarker, 2015; Zhou, Huddleston, & Lundy, 1992). The deflection measurements are then used in what is known as back-calculation of the multi-layered linear elastic theory to yield the effective (i.e. existing) pavement layer moduli, which are used to estimate the pavement remaining strength. The second approach is an empirical one known as the effective thickness approach or component analysis method which attempts to estimate the remaining strength using equivalency conversion factors/correction factors that are assigned based on the outcomes of pavement distress surveys (Abaza, 2005, 2018; American Association of State Highway and Transportation Officials [AASHTO], 1993; Asphalt Institute [AI], 1996; Bianchini, Gonzalez, & Bell, 2018; Huang, 2004).

However, several researchers had developed modified approaches to estimate the pavement remaining strength and relevant resurfacing thickness. Abaza (2005) proposed performance-based models to estimate the effective structural capacity with equivalency conversion factors derived from the performance curves developed using the AASHTO-based deterministic prediction model. Zhou, Hu, and Scullion (2010) presented a comprehensive mechanistic-empirical system to design a balanced asphaltic overlay based on traffic loadings, climate conditions, existing pavement conditions, and material properties of asphalt overlay mix. Maji, Singh, and Chawla (2016) developed a comprehensive probabilistic approach for asphaltic overlay design that can accommodate variations in the design parameters which include layer thicknesses, layer moduli, vehicle damage factor, lane distribution factor, and traffic growth rate. Nobakht, Sakhaeifar, and Newcomb (2016) proposed a rehabilitation strategy based on structural capacity, which mainly applies a damage ratio estimated from the (FWD) data and a rutting index estimated from distress assessment. Le, Lee, Flores, Baek, and Park (2017) developed a simple regression model for estimating overlay design thickness based on the mechanistic-empirical approach. The model is a function of the layer thicknesses, asphaltic modulus ratio, subgrade condition, and traffic volume. Bianchini et al. (2018) proposed a modified overlay procedure that accounts for the structural condition of the existing asphaltic surface mainly using an asphaltic correction factor estimated from the existing load-related distresses. Abaza (2018) proposed an Empirical-Markovian model to predict the overlay design thickness as a function of the original

structural capacity and other related parameters including pavement deterioration transition probabilities.

The vast majority of the previously outlined resurfacing approaches mainly deal with estimating the present pavement remaining strength as they are not designed to predict the future remaining strength, which is a key requirement for pavement management applications. Therefore, the main objective of this research paper is to develop a simplified empirical approach that can predict the future remaining strength and consequently relevant resurfacing thickness while taking into consideration the two main design factors, namely asphaltic strength loss and future traffic growth. The detailed research objectives are as described below:

- (1) Developing a resurfacing procedure for flexible pavement that deploys potential key performance indicators (KPIs) as the main input parameters. The KPIs are readily available to highway agencies including local governments.
- (2) Developing simplified empirical models that can effectively predict the remaining strength factors for both thin and thick asphaltic surfaces to be used in estimating the resurfacing thickness component due to asphaltic strength loss.
- (3) Developing a simplified calibration procedure to effectively calibrate the remaining strength factor models using minimization of the sum of squared errors (SSE) with the error being defined as the difference between the estimated resurfacing thickness and observed resurfacing thickness.
- (4) Proposing a simplified procedure to estimate the resurfacing thickness component due to future traffic growth using empirical design methods of flexible pavement.

## 2. Methodology

This section presents the simplified empirical approach for predicting the remaining strength factor used in the rehabilitation of flexible pavement. It also presents the mathematical models used to estimate the asphaltic resurfacing thicknesses as applied to the two main rehabilitation strategies, namely plain overlay, and cold milling and overlay. The models are developed mainly relying on the use of empirical methods for flexible pavement design. Generally, the asphaltic resurfacing thickness has to account for two design factors: the first one is anticipated future increases in traffic load applications, and the second one is the pavement strength degradation as a result of the progressive traffic load action. Therefore, the asphaltic resurfacing thickness has two components as outlined next.

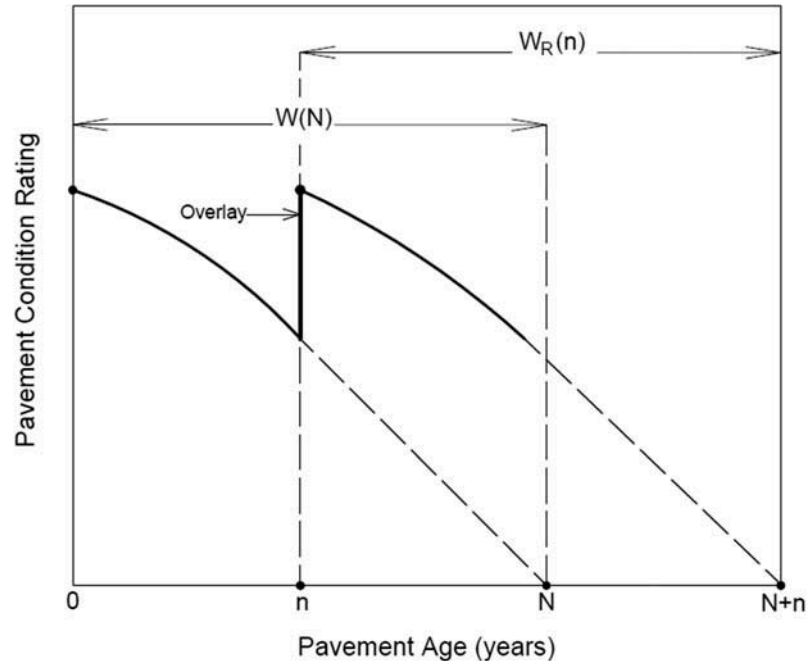
### 2.1. Resurfacing thickness component due to traffic growth

The first component of the resurfacing thickness is to be estimated assuming that pavement rehabilitation will be scheduled at pavement age of (n) years as shown in Figure 1. The rehabilitated pavement structure is to be designed to withstand the traffic load applications that are expected to take place over an analysis period of (N) years, which is the same as the analysis period used in the design of original pavement. The resurfacing thickness associated with this component,  $h_1(n)$ , is computed using Equation (1). The structural capacity associated with this component is estimated as the difference between the structural capacity required for rehabilitated pavement,  $SC(n)$ , and the corresponding one associated with original pavement,  $SC(0)$ . This difference is divided by a relative strength coefficient,  $s_{AC}(n)$ , to yield the equivalent asphaltic thickness as normally deployed by the empirical design methods such as the AASHTO and Caltrans methods (AASHTO, 1993; California Department of Transportation, Caltrans, 2008).

$$h_1(n) = \frac{SC(n) - SC(0)}{s_{AC}(n)} \quad (1)$$

The structural capacity for original pavement,  $SC(0)$ , is estimated using the 80 kN equivalent single axle load (ESAL) applications,  $W(N)$ , expected to travel the pavement over an analysis period of (N) years as shown in Figure 1. Similarly, the structural capacity for rehabilitated pavement,  $SC(n)$ , is to

**Figure 1. Typical pavement rehabilitation scheduling plan.**



be estimated using the 80 kN ESAL applications,  $W_R(n)$ , expected to travel the pavement over the same analysis period of  $(N)$  years. Equation (2) can be used to calculate the  $W_R(n)$  based on the first-year 80 kN ESAL applications,  $W(1)$ , and relevant traffic growth factors, namely  $GF(N + n)$  and  $GF(n)$ . The relevant growth factors are computed using the formula proposed by the Asphalt Institute (AI 1999), which mainly requires the annual traffic growth rate ( $r$ ) in decimal form, and the corresponding analysis period in years.

$$W_R(n) = W(1) [GF(N + n) - GF(n)] \quad (2)$$

where:

$$W(1) = W(N)/GF(N)$$

$$GF(n) = \frac{(1 + r)^n - 1}{r}$$

The first-year 80 kN ESAL applications,  $W(1)$ , is determined from dividing the design 80 kN ESAL applications,  $W(N)$ , associated with original pavement by the growth factor,  $GF(N)$ , corresponding to an analysis period of  $(N)$  years.

## 2.2. Resurfacing thickness component due to strength loss

The second component of the resurfacing thickness,  $h_2(n)$ , is computed using Equation (3), which mainly accounts for the strength losses endured by the pavement structure over a service life of  $(n)$  years. The structural capacity for the second component is estimated as the difference between the structural capacity of the original pavement,  $SC(0)$ , and the effective structural capacity associated with the existing pavement,  $SC_{eff}(n)$ , at the time of rehabilitation. Again, this difference is divided by a relative strength coefficient,  $s_{AC}(n)$ , to obtain the corresponding asphaltic resurfacing thickness.

$$h_2(n) = \frac{SC(0) - SC_{eff}(n)}{s_{AC}(n)} \quad (3)$$

The structural capacity associated with original pavement,  $SC(0)$ , is typically determined as the product sum of the layer relative strength coefficients,  $s_i(0)$ , and the corresponding layer thicknesses,  $D_i$ , as defined in Equation 4(a). Similarly, Equation 4(b) is used to calculate the effective structural capacity,  $SC_{eff}(n)$ , mainly relying on the effective relative strength coefficients,  $s_i(n)$ , which are expected to be lower in values compared to the original coefficient values, thus accounting for the layer strength losses.

$$SC(0) = \sum_i s_i(0) \times D_i \quad (4a)$$

$$SC_{eff}(n) = \sum_i s_i''(n) \times D_i \quad (4b)$$

However, it has been reported that the asphaltic surface layer is the main layer that endures substantial strength loss over time, while underlying pavement layers suffer minor strength losses which can be neglected when calculating the effective structural capacity (Abaza, 2005, 2018; Bianchini et al., 2018; Huang, 2004). Therefore, Equation (5) is mainly a reproduction of Equation (3) but assuming the asphaltic surface layer is the only layer enduring strength loss over time. In essence, Equation (5) attempts to compensate the existing asphaltic layer for any strength loss it has suffered over a service life of  $(n)$  years.

$$h_2(n) = \frac{SC_{AC}(0) - s_{AC}''(n) \times D_{AC}}{s_{AC}(n)} \quad (5)$$

Equation (5) assumes that the deployed rehabilitation strategy mainly consists of plain overlay. However, in the case of cold milling and overlay, the second component of the resurfacing thickness,  $h_2(n)$ , is computed using Equation (6) wherein the cold milling thickness ( $D_m$ ) is subtracted from the existing asphaltic thickness ( $D_{AC}$ ) to yield the effective asphaltic thickness (i.e. remaining asphaltic thickness after cold milling).

$$h_2(n) = \frac{s_{AC}(0) \times D_{AC} - s_{AC}''(n) \times (D_{AC} - D_m)}{s_{AC}(n)} \quad (6)$$

The effective asphaltic relative strength coefficient,  $s_{AC}''(n)$ , can be determined based on both destructive and non-destructive testing of the pavement structure (Hong, 2014; Huang, 2004; Jimoh, Itiola, & Afolabi, 2015). State highway agencies typically use the falling-weight deflectometer (FWD) procedure to obtain an estimate of the effective relative strength coefficient as a function of the effective resilient modulus, which is normally estimated from the back-calculation of the multi-layered linear elastic theory (Gedafa, Hossain, Romanoschi, & Gisi, 2010; Hoffman, 2003; Sarker et al., 2015; Smith et al., 2017). However, local governments do not typically have access to the deflection instruments needed to carry out the FWD procedure; therefore this paper proposes a simplified approach to estimate the effective asphaltic relative strength coefficient as outlined in Equation (7). It is simply defined as the product of the original asphaltic relative strength coefficient,  $s_{AC}(0)$ , and the asphaltic remaining strength factor,  $F(n)$ .

$$s_{AC}''(n) = s_{AC}(0) \times F(n) \quad (7)$$

The asphaltic remaining strength factor,  $F(n)$ , is to be mainly estimated using reliable key performance indicators (KPIs) as defined in the subsequent section. Equation (8) is derived when substituting in Equation (6) the value of the effective relative strength coefficient as defined in Equation (7) and assuming the original relative strength coefficient,  $s_{AC}(0)$ , is equal to the corresponding value associated with rehabilitated pavement,  $s_{AC}(n)$ .

$$h_2(n) = D_{AC} - F(n) \times (D_{AC} - D_m) \quad (8)$$

The total resurfacing thickness,  $h(n)$ , at the rehabilitation time of  $(n)$  years is the sum of the two resurfacing components,  $h_1(n)$  and  $h_2(n)$ , thus accounting for both future increases in traffic load applications and strength degradation due to traffic load action.

### 2.3. Estimation of asphaltic remaining strength factor

The asphaltic remaining strength factor,  $F(n)$ , can be estimated using key performance indicators (KPIs) as outlined in Equation (9). The well-known KPIs include the present serviceability index (PSI), international rough index (IRI), and pavement condition index (PCI). The PSI is typically estimated based on the roadway profile roughness, and pavement cracking and deformation (AASHTO, 1993; Huang, 2004). It is highly correlated to the IRI which is mainly defined as a function of the roadway longitudinal profile roughness (Al-Omari & Darter, 1994; Hall & Muñoz, 1999; Sayers, 1995). The PCI is mainly estimated based on the prevailing pavement defects as obtained from field surveys and does not directly account for the roadway profile roughness (Shahin, 1994). Equation 9(a) can be used to estimate the remaining strength factor when considering KPIs with present value ( $KPI_t$ ) is typically lower than the initial one ( $KPI_o$ ), which is the case when using PSI and PCI. Similarly, Equation 9(b) can be used when considering IRI as the KPI since the present value ( $IRI_t$ ) is typically higher than the initial one ( $IRI_o$ ).

$$F(n) = \left( \frac{KPI(n)}{KPI(0)} \right)^K = \left( \frac{KPI_t}{KPI_o} \right)^K \quad KPI_t \leq KPI_o \quad (9a)$$

$$F(n) = \left( \frac{KPI(0)}{KPI(n)} \right)^K = \left( \frac{KPI_o}{KPI_t} \right)^K \quad KPI_t \geq KPI_o \quad (9b)$$

The value of the remaining strength factor as indicated by Equation (9) is lower than one, a necessary requirement for estimating the effective relative strength coefficient defined in Equation (7). A calibration constant ( $K$ ) is introduced in Equation (9) to be estimated from the calibration procedure with the optimal ( $K$ ) value is to be derived from the minimization of the sum of squared errors (SSE) as presented later. According to Equation (9), the present KPI value can be replaced by the predicted future one,  $KPI(n)$ , for pavement management applications.

Pavement performance prediction has been performed using both deterministic and probabilistic approaches (Abaza, 2004; Amin, 2015; Li, Haas, & Xie, 1997). The outcome in both cases is the development of simple mathematical models that enable pavement engineers to estimate the future pavement conditions in terms of reliable KPIs such as the PSI and PCI. Generally, two distinct types of pavement performance have been identified by researchers (Abaza, 2004; Chang & Ramirez-Flores, 2015; Garber & Hoel, 2014). The first type is superior performance defined in Equation 10(a) as a polynomial with 2nd degree, but is associated with increasingly higher deterioration rates. The second type is inferior performance as presented in Equation 10(b), but is associated with decreasingly lower deterioration rates. Models like the ones presented in Equation (10) can be used to estimate the future KPI values for pavement management when considering KPIs that decrease over time such as the PSI and PCI.

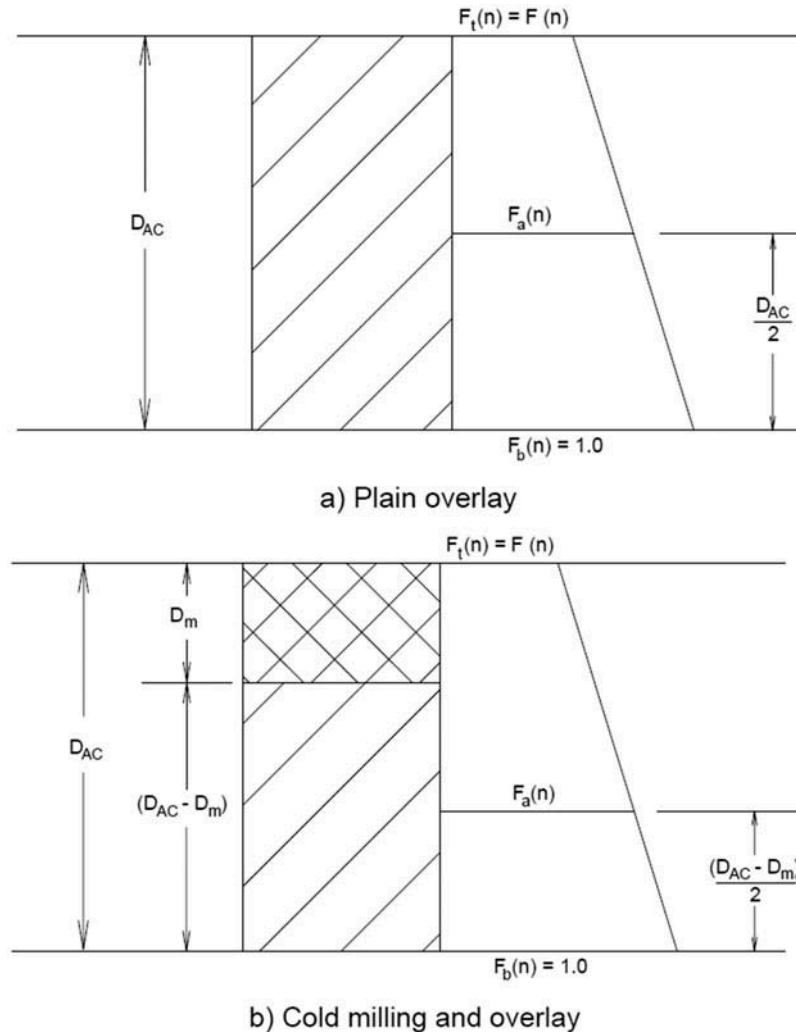
$$KPI(n) = -A_s n^2 - B_s n + C_s \quad (10a)$$

$$KPI(n) = A_i n^2 - B_i n + C_i \quad (10b)$$

The remaining strength factor as defined in Equation (9) is more applicable to thin asphaltic surface layers as potential KPIs mainly focus on evaluating the pavement defects prevailing at the top of the asphaltic surface and extending downward to almost covering the full depth of the thin asphaltic layer. However, the prevailing pavement defects may not practically extend downward to cover the full depth of a thick asphaltic layer; therefore, it is proposed in the case of thick asphaltic surface to use an average remaining strength factor,  $F_a(n)$ . Figure 2 shows a representation of the two potential rehabilitation strategies, namely plain overlay ( $D_m = 0$ ), and cold milling and overlay assuming thick asphaltic surface. In both strategies, it is assumed that the remaining strength factor as estimated using Equation (9) only represents the pavement distress condition at the top asphaltic fiber [i.e.  $F_t(n) = F(n)$ ], while its value at the bottom asphaltic fiber is equal to one [i.e.  $F_b(n) = 1$ ]. The average remaining strength factor,  $F_a(n)$ , in the case of plain overlay is determined as the average value of the two corresponding values associated with the top and bottom asphaltic fibers as presented in Equation (11).



**Figure 2. Estimation of average remaining strength factor for thick asphaltic surface.**



$$F_a(n) = \frac{1 + F(n)}{2} \quad (11)$$

Similarly, in the case of cold milling and overlay, the average remaining strength factor is estimated based on linear interpolation so that it represents the asphaltic remaining strength at the middle of the remaining asphaltic thickness,  $(D_{AC} - D_m)/2$ , as shown in Figure 2. The outcome of the linear interpolation is the derivation of Equation (12), which can be used to compute the average remaining strength factor for thick asphaltic surface,  $F_a(n)$ , as a function of the existing asphaltic surface thickness ( $D_{AC}$ ), cold milling thickness ( $D_m$ ), and the remaining strength factor for thin asphaltic surface,  $F(n)$ , as obtained from Equation (9). Equation (12) becomes equivalent to Equation (11) in the case of plain overlay ( $D_m = 0$ ).

$$F_a(n) = 1 - \left( \frac{D_{AC} - D_m}{2} \right) \left( \frac{1 - F(n)}{D_{AC}} \right) \quad (12)$$

The second component of the resurfacing thickness,  $h_2(n)$ , due to asphaltic strength loss can now be calculated using Equation (13) when considering thick asphaltic surface. Equation (13) is similar to Equation (8) with the exception of using the average remaining strength factor,  $F_a(n)$ , instead of the remaining strength factor,  $F(n)$ , for thin asphaltic surface.

$$h_2(n) = D_{AC} - F_a(n) \times (D_{AC} - D_m) \quad (13)$$

Equations (14) and (15) provide a summary of the simple derived models for estimating the second component of the resurfacing thickness,  $h_2(n)$ , considering thin and thick asphaltic surfaces, respectively, for both plain overlay ( $D_m = 0$ ), and cold milling and overlay.

$$h_2(n) = \begin{cases} D_{AC}(1 - F(n)), D_m = 0 \\ D_{AC} - F(n)(D_{AC} - D_m) \end{cases} \quad (14)$$

$$h_2(n) = \begin{cases} D_{AC}(1 - F_a(n)), F_a(n) = \frac{1+F(n)}{2}, D_m = 0 \\ D_{AC} - F_a(n)(D_{AC} - D_m), F_a(n) = 1 - \left(\frac{D_{AC}-D_m}{2}\right) \left(\frac{1-F(n)}{D_{AC}}\right) \end{cases} \quad (15)$$

#### 2.4. Calibration of remaining strength factor model

The calibration of the model presented in Equation (9) for the purpose of estimating the model exponent (K) can be performed using minimization of the sum of squared errors (SSE). The error is defined as the difference between the estimated (predicted) resurfacing thickness and observed (provided) resurfacing thickness. The minimization procedure of SSE as defined in Equation (16) requires a representative sample of pavement rehabilitation projects with size (m), and the corresponding estimated resurfacing thicknesses,  $h_e(j)$ , and observed resurfacing thicknesses,  $h_o(j)$ , as actually provided. The estimated resurfacing thicknesses are as obtained from the simplified empirical approach presented in this paper, whereas the observed (i.e. provided) resurfacing thicknesses are as derived from a reliable overlay design procedure.

$$\text{Minimize : SSE} = \sum_{j=1}^m [h_e(j) - h_o(j)]^2 \quad (16)$$

The estimated resurfacing thickness,  $h_e(j)$ , is represented by the second component of the asphaltic resurfacing thickness,  $h_2(j)$ , as indicated by Equation (17) since it is a function of the calibration constant (K). The first component of resurfacing thickness,  $h_1(j)$ , can later be added to the observed resurfacing thickness to obtain the total resurfacing thickness,  $h(j)$ , for the  $j$ th rehabilitation project. The  $h_1(j)$  component is independent of the calibration constant (K).

$$h_e(j) = h_2(j) = \begin{cases} D_{AC}(j) - F(j) \times [D_{AC}(j) - D_m(j)], \text{Thin asphalt} \\ D_{AC}(j) - F_a(j) \times [D_{AC}(j) - D_m(j)], \text{Thick asphalt} \end{cases} \quad (17)$$

where:

$$F(j) = \begin{cases} \left(\frac{KPI_t(j)}{KPI_o(j)}\right)^K & KPI_t(j) \leq KPI_o(j) \\ \left(\frac{KPI_o(j)}{KPI_t(j)}\right)^K & KPI_t(j) \geq KPI_o(j) \end{cases}$$

$$F_a(j) = 1 - \left(\frac{D_{AC}(j) - D_m(j)}{2}\right) \left(\frac{1 - F(j)}{D_{AC}(j)}\right)$$

The minimization of SSE as indicated by Equations (16) and (17) can easily be programmed using the "Excel" software package available in Microsoft Word or any other software packages such as Matlab. A trial and error exhaustive-search approach can be initiated by incrementally varying the (K) value using a hundredth point increment until reaching the optimal (K) value that is associated with the minimal SSE value. According to the sample problems presented later, the typical (K) range is (0.5–4.0), therefore only a few hundreds of search evaluations are needed.

#### 2.5. Application of empirical pavement design methods

The estimation of the first component of resurfacing thickness,  $h_1(n)$ , due to future increases in traffic load applications requires the use of a potential empirical pavement design method. This section reviews the two most popular empirical methods for flexible pavement design, namely AASHTO and Caltrans, which can be used to estimate the first component thickness as defined in



Equation (1). The AASHTO design method deploys the structural number (SN) to define the pavement structural capacity (SC), and the asphaltic layer coefficient ( $a_{AC}$ ) to represent the asphaltic relative strength coefficient ( $s_{AC}$ ) (AASHTO, 1993). Therefore, Equation (1) is revised as indicated by Equation (18) to comply with the AASHTO design requirements (AASHTO, 1993). A unit conversion factor is provided in Equation (18) to convert thickness from inches to centimeters. The typical value for the asphaltic layer coefficient ( $a_{AC}$ ) is (0.44) assuming hot-mix asphalt with high stability (AASHTO, 1993).

$$h_1(n) = 2.5 \left( \frac{SN(n) - SN(0)}{a_{AC}(n)} \right) \quad (18)$$

The design structural number (SN) is to be derived from the AASHTO basic design equation using relevant design parameters as indicated by Equation (19). The SN(n) at the rehabilitation time of (n) years is to be computed from Equation (19) using a trial and error approach or by consulting the AASHTO equivalent design chart. The  $W_R(n)$  is determined as indicated by Equation (2), which represents the 80 kN ESAL applications expected to travel the rehabilitated pavement over an analysis period comprised of (N) years as shown in Figure 1.

$$\log W_R(n) = Z_R S_o + 9.36 \log(SN(n) + 1) + \frac{\log \left[ \frac{\Delta PSI}{4.2 - 1.5} \right]}{0.40 + \frac{1094}{(SN(n) + 1)^{5.19}}} + 2.32 \log(M_R) - 8.27 \quad (19)$$

where:  $W_R(n)$  = 80 kN ESAL applications associated with an analysis period of (N) years as shown in Figure 1,

$Z_R$  = standard normal deviate for a specified reliability level,

$S_o$  = combined standard error of the traffic prediction and performance prediction,

$\Delta PSI$  = difference between the initial or present serviceability index and the terminal serviceability index,

$SN(n)$  = design structural number associated with an analysis period (N), and

$M_R$  = subgrade resilient modulus and must be in pound per square inch.

The Caltrans design method deploys the gravel equivalent (GE) to describe the pavement structural capacity (SC), and the gravel equivalent factor ( $Gf_{AC}$ ) to denote the asphaltic relative strength coefficient ( $s_{AC}$ ) (California Department of Transportation, Caltrans, 2008). Equation (20) is a reproduction of Equation (1) but revised to be in line with the Caltrans design requirements. A unit conversion factor is provided in Equation (20) to convert thickness from feet to centimeters. The Caltrans design manual recommends a gravel equivalent factor ( $Gf_{AC}$ ) of (1.9) when considering hot-mix asphalt overlays (California Department of Transportation, Caltrans, 2008).

$$h_1(n) = 30 \left( \frac{GE(n) - GE(0)}{Gf_{AC}(n)} \right) \quad (20)$$

The Caltrans design method requires the use of Equation (21) to calculate the design gravel equivalent,  $GE(n)$ , as a function of the design traffic index,  $TI(n)$ , and subgrade resistance value ( $R_s$ ). The design traffic index is determined based on the 80 kN ESAL applications,  $W_R(n)$ , expected to travel the rehabilitated pavement over an analysis period comprised of (N) years as shown in Figure 1, which also shows that (n) is the rehabilitation scheduling time in years.

$$GE(n) = 0.0032 \times TI(n) \times (100 - R_s) \quad (21)$$

where:

$$TI(n) = 9.0 \times \left( \frac{W_R(n)}{10^6} \right)^{0.119}$$

The estimation of the second component of resurfacing thickness,  $h_2(n)$ , due to asphaltic strength loss is independent of using any pavement design method. It is only dependent on the asphaltic remaining strength factor,  $F(n)$ , existing asphaltic layer thickness ( $D_{AC}$ ), and cold milling thickness ( $D_m$ ) as presented in Equations (14) and (15) considering a rehabilitation scheduling time of ( $n$ ) years. The subsequent sample presentation section shows that the second component thicknesses are typically higher in values compared to the first component thicknesses.

### 3. Sample presentation

This section presents two case studies to demonstrate the use of the presented simplified empirical approach for predicting the asphaltic remaining strength factor as applied to pavement rehabilitation. The first case study predicts the asphaltic remaining strength factor for two sample rehabilitation projects with known future pavement performance, and also estimates the corresponding resurfacing thicknesses considering both plain overlay, and cold milling and overlay. The second case study focuses on the calibration of the remaining strength factor model for a local roadway sample deploying the three potential key performance indicators, namely PSI, IRI, and PCI.

#### 3.1. Case study I: prediction of remaining strength factor

Two pavement projects (A & B) have been considered for estimating the two-component thicknesses associated with resurfacing by deploying the simplified empirical approach presented in this paper. The design input data provided in Table 1 has been used to estimate the first component of resurfacing thickness,  $h_1(n)$ , due to traffic growth with the results provided in Table 2. The 80 kN ESAL applications,  $W_R(n)$ , associated with rehabilitated pavement has been determined using Equation (2) assuming 20-year analysis period ( $N$ ) and two different values of the annual traffic growth rate ( $r = 3\% \text{ \& } 6\%$ ). The design structural number for rehabilitated pavement,  $SN(n)$ , has been computed using Equation (19) with the corresponding resurfacing thickness,  $h_1(n)$ , calculated from Equation (1). The first component thickness is relatively small in magnitude (0.40–0.81 cm) as the rehabilitation time (i.e. service time) increases from 6 to 12 years in the case of superior performance (Project A) considering 3% traffic growth rate. The thicknesses become about 50% higher (0.58–1.20 cm) in the case of inferior performance (Project B) assuming 3% traffic growth rate. Table 2 also indicates that the first component thicknesses almost double in values when the traffic growth rate increased to 6%, and that the first component thickness is almost linearly proportional to the rehabilitation scheduling time ( $n$ ) considering both Projects (A & B).

Abaza (2004) predicted the future performance of Projects (A & B) using the design input data provided in Table 1 and the AASHTO-based deterministic prediction approach. The pavement performance of Project (A) was recognized as a superior one with the corresponding prediction model as indicated by Equation 22(a), which is equivalent to Equation 10(a). Similarly, Project (B) performance was identified as inferior with the relevant prediction model as provided in Equation 22(b), which is comparable to Equation 10(b). Equations 22(a) and 22(b) have been used to

**Table 1. Basic pavement design data according to AASHTO method for two sample rehabilitation projects ( $N = 20$  years,  $a_{AC}(0) = 0.44$ )**

Project	$W(N) \times 10^6$	$MR_s \times 10^3$	$Z_R$	$S_o$	$\Delta PSI$	$SN(0)$	$MR_b \times 10^3$	$SN_{AC}(0)$	$D_{AC}$ (cm)
A <sup>a</sup>	1.0	15	−1.645	0.35	2.5	2.57	30	2.0	11.5
B <sup>b</sup>	1.0	3	−1.645	0.35	2.5	4.38	30	2.0	11.5

<sup>a</sup> Superior performance

<sup>b</sup> Inferior performance

**Table 2. Sample thickness calculations of the first resurfacing component,  $h_1(n)$ , due to traffic growth ( $a_{AC}(n) = 0.44$ )**

n (yrs.)	Project A			Project B		Project A			Project B	
	$W_R(n)^a$ ( $\times 10^6$ )	SN(n)	$h_1(n)$ (cm)	SN(n)	$h_1(n)$ (cm)	$W_R(n)^b$ ( $\times 10^6$ )	SN(n)	$h_1(n)$ (cm)	SN(n)	$h_1(n)$ (cm)
6	1.193	2.641	0.40	4.480	0.58	1.419	2.711	0.80	4.585	1.18
7	1.229	2.653	0.47	4.498	0.68	1.505	2.734	0.94	4.621	1.39
8	1.266	2.665	0.54	4.516	0.79	1.595	2.758	1.07	4.657	1.60
9	1.304	2.677	0.61	4.534	0.89	1.690	2.782	1.21	4.692	1.80
10	1.344	2.689	0.68	4.552	0.99	1.792	2.806	1.35	4.729	2.01
11	1.383	2.700	0.74	4.569	1.09	1.899	2.830	1.49	4.765	2.22
12	1.425	2.712	0.81	4.588	1.20	2.013	2.855	1.63	4.802	2.44

<sup>a</sup> 80 KN ESAL applications assuming 3% annual traffic growth rate (r).

<sup>b</sup> 80 KN ESAL applications assuming 6% annual traffic growth rate (r).

**Table 3. Sample thickness calculations of the second resurfacing component,  $h_2(n)$  in centimeters, due to strength loss for Project A ( $PSI_0 = 4.5$ ,  $K = 0.8$ )**

n (yrs.)	PSI(n)	$D_m$ (cm)	Thin Asphalt			Thick Asphalt		
			F(n)	$h_2(n)$	h(n)	$F_a(n)$	$h_2(n)$	h(n)
6	4.155	0	0.938	0.71	1.11	0.969	0.36	0.76
7	4.049	0	0.919	0.93	1.40	0.960	0.46	0.93
8	3.929	0	0.897	1.18	1.72	0.948	0.60	1.14
9	3.795	0	0.873	1.46	2.07	0.936	0.74	1.35
10	3.647	0	0.845	1.78	2.46	0.922	0.90	1.58
11	3.485	0	0.815	2.13	2.87	0.908	1.06	1.80
12	3.309	0	0.782	2.51	3.32	0.891	1.25	2.06

**Table 4. Sample thickness calculations of the second resurfacing component,  $h_2(n)$  in centimeters, due to strength loss for Project B ( $PSI_0 = 4.5$ ,  $K = 1.5$ )**

n (yrs.)	PSI(n)	$D_m$ (cm)	Thin Asphalt			Thick Asphalt		
			F(n)	$h_2(n)$	h(n)	$F_a(n)$	$h_2(n)$	h(n)
6	3.396	0	0.656	3.96	4.54	0.828	1.98	2.56
7	3.254	0	0.615	4.43	5.11	0.808	2.21	2.89
8	3.118	0	0.577	4.86	5.65	0.788	2.44	3.23
9	2.988	2	0.541	5.28	6.17	0.770	4.18	5.07
10	2.864	2	0.508	5.66	6.65	0.754	4.34	5.33
11	2.746	3	0.477	6.01	7.10	0.738	5.23	6.32
12	2.634	3	0.448	6.35	7.55	0.724	5.35	6.55

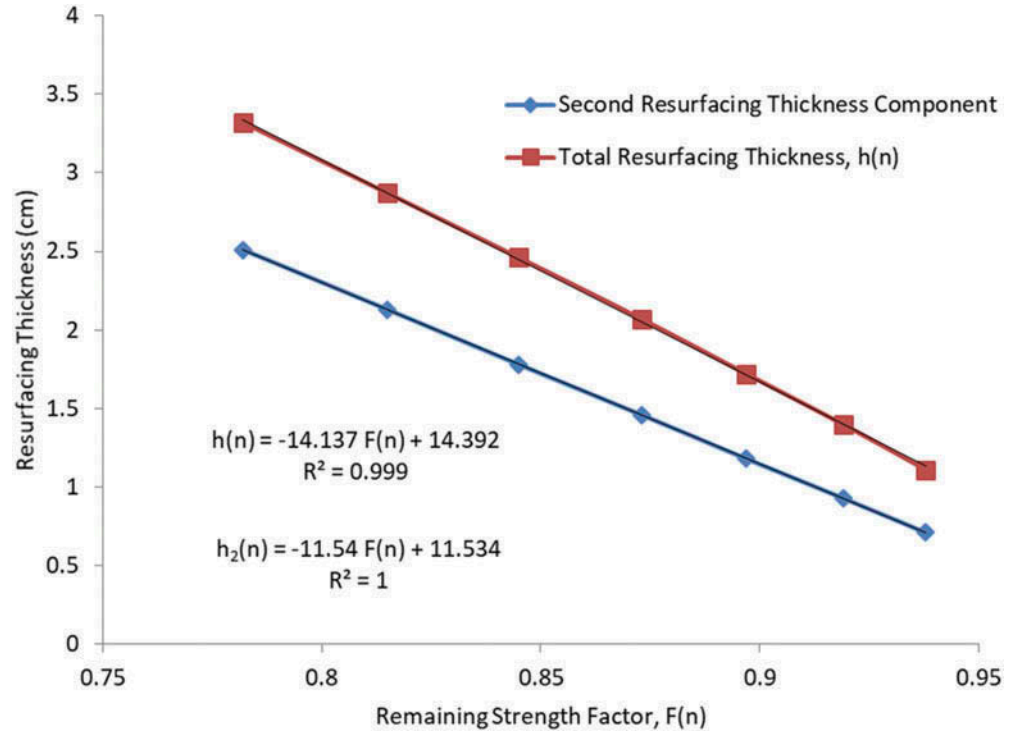
estimate the PSI as a function of the rehabilitation scheduling time (n), in years, with the results provided in Tables 3 and 4 for Projects (A & B), respectively. The PSI values for Project (B) are substantially lower than the corresponding ones associated with Project (A) considering the same rehabilitation times (n), an indication of inferior performance.

$$PSI(n) = -0.007n^2 - 0.015n + 4.497 \quad (22a)$$

$$PSI(n) = 0.003n^2 - 0.181n + 4.374 \quad (22b)$$

The remaining strength factors, F(n) and  $F_a(n)$ , have been estimated for Projects (A & B) using Equations 9(a) and (12) by assuming both thin and thick asphaltic surfaces, respectively. The asphaltic surface thickness ( $D_{AC}$ ) associated with Projects (A & B) is 11.5 cm, which has been treated as both thin and thick surface for sample presentation purposes. The definition of thin/thick surface is left to professional experience and engineering judgment, but the author recommends a minimum of 5 inches (12 cm) to be considered as a thick asphaltic surface. The second component of the resurfacing thickness,  $h_2(n)$ , due to strength loss has been determined using Equations (14) and (15) with relevant results provided in Tables 3 and 4 for Projects (A & B), respectively. The cold milling thickness ( $D_m$ ) is only specified when the PSI drops below 3.0, and it starts with a minimum of 2 cm and assumed to increase by 1 cm with each decrease of 0.25 PSI increment. The calibration factor (K) has been assigned the values of (0.8 & 1.5) for Projects (A & B), respectively, with the initial PSI value,  $PSI(0)$ , assumed to be 4.5. Tables 3 and 4 also provide the total resurfacing thickness, h(n), with the first component,  $h_1(n)$ , taken as presented in Table 2 with 3% annual traffic growth rate.

**Figure 3. Resurfacing thickness as a function of remaining strength factor for Project (A) assuming thin asphaltic surface.**



**Figure 4. Resurfacing thickness as a function of average remaining strength factor for Project (A) assuming thick asphaltic surface.**

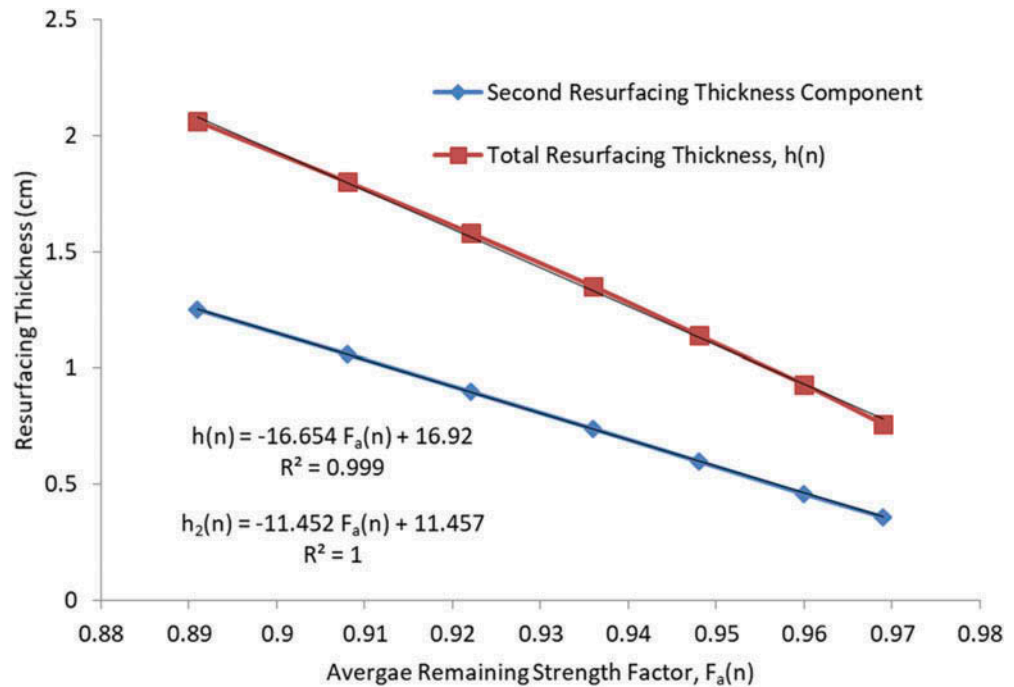


Table 3 indicates the range for the second component thickness,  $h_2(n)$ , to be (0.71–2.51 cm) for Project (A) when assuming thin asphaltic surface, but this range is reduced by half when assuming thick asphaltic surface (0.36–1.25 cm) considering plain overlay ( $D_m = 0.0$ ). The ranges for the total resurfacing thickness,  $h(n)$ , are (1.11–3.32 cm) and (0.76–2.06 cm) for thin and thick asphaltic surfaces, respectively, considering superior performance. Figures 3 and 4 indicate a perfect linear

inverse relationship ( $R^2 = 1$ ) between the remaining strength factor and resurfacing thickness based on the results provided in Table 3. Similarly, Table 4 provides the ranges for  $h_2(n)$  to be (3.96–6.35 cm) and (1.98–5.35 cm) assuming thin and thick asphaltic surfaces, respectively, for Project (B) with inferior performance. Again, the ratio of  $h_2(n)$  for thick surface to the corresponding value for thin surface remains at (0.5) when considering plain overlay; however, this ratio increases up to (0.87) when cold milling is applied. This generally indicates that as the cold milling thickness increases, the  $h_2(n)$  value for thick surface gets closer to the corresponding value for thin surface. The total resurfacing thicknesses associated with inferior performance (Table 4) are substantially higher than the corresponding values for superior performance (Table 3) as one would expect. Also, the second component thicknesses represent considerable portions of the total resurfacing thicknesses in the case of inferior performance. The specified (K) values seem to be appropriate for the two investigated projects; however, the next section provides sample results for estimating the optimal (K) value for a local roadway sample.

### 3.2. Case study II: calibration of remaining strength factor model

A sample of 12 local roadways with low volume has been considered for the purpose of calibrating the remaining strength factor model presented in Equations (16) and (17). The selected roadway sample consists of two-lane rural roads providing villages in the northern district of Nablus, Palestine, with access to nearby main highways. Table 5 provides the pavement basic design parameters for the local roadway sample according to Caltrans method which includes the asphaltic surface thickness ( $D_{AC}$ ), aggregate base thickness ( $D_b$ ), original design ESAL,  $W(N)$ , and subgrade resistance value ( $R_s$ ). Based on the original design ESAL, the selected roadway sample can be classified as essentially consisting of low volume roads. These roads were initially reconstructed about 10–12 years ago, and they are now considered for rehabilitation. The present PSI value ( $PSI_t$ ) for each roadway has been estimated using the normal procedure recommended by AASHTO with the results provided in Table 7.

The common practice followed by highway agencies is to use key performance indicators (KPIs) such as PSI, IRI, and PCI to propose pavement rehabilitation strategies and consequently specify relevant resurfacing thicknesses. Table 6 provides an example of relating the observed resurfacing thickness ( $h_o$ ) to potential KPIs as typically implemented by local governments. The PSI ranges provided in Table 6 are converted to their equivalent IRI ranges using Equation (23) proposed by Al-Omari and Darter (1994). Equation (23) can be used to estimate the PSI from IRI and vice versa with the IRI is in the unit of (m/km). Alternatively, the IRI can be field estimated using instruments such as the Profilograph. Table 6 also provides the equivalent PCI ranges as estimated by the author deploying experience and engineering judgment. Table 8 provides for each roadway the equivalent IRI value computed using Equation (23) based on the corresponding PSI value provided in Table 7. Similarly, Table 9 provides the equivalent PCI values as estimated from field surveys of pavement distress.

$$PSI = 5e^{(-0.24IRI)} \quad (23)$$

The observed resurfacing thicknesses,  $h_o(j)$ , for thin asphaltic surface as provided in Tables 7–9 are obtained from Table 6 based on the three deployed KPIs and have been used in the minimization procedure of SSE outlined in Equations (16) and (17). Tables 7–9 also present a case of thick asphaltic surface wherein the corresponding asphaltic thicknesses,  $D_{AC}(j)$ , are attained by adding

**Table 5. Pavement basic design parameters for the local roadway sample**

Road j	1	2	3	4	5	6	7	8	9	10	11	12
$D_{AC}$ (cm)	8	8	8	7	7	8	8	8	9	8	9	9
$D_b$ (cm)	25	25	30	30	35	30	35	40	45	35	40	45
$W(N) \times 10^3$	470	320	350	180	250	520	650	700	620	580	660	750
$R_s$	46	41	30	27	24	40	28	21	10	26	16	13



**Table 6. Sample resurfacing thickness by prescription method for local roads**

<b>PSI</b>	<b>3.50–3.25</b>	<b>3.25–3.00</b>	<b>3.00–2.75</b>	<b>2.75–2.50</b>	<b>2.50–2.25</b>	<b>2.25–2.00</b>	<b>2.00–1.75</b>	<b>1.75–1.50</b>
IRI (m/km)	1.49–1.79	1.79–2.13	2.13–2.49	2.49–2.89	2.89–3.33	3.33–3.82	3.82–4.37	4.37–5.02
PCI	75–70	70–65	65–60	60–55	55–50	50–45	45–40	40–35
h <sub>o</sub> (cm)	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0

**Table 7. Sample resurfacing thickness using present serviceability index (PSI) as a key performance indicator ( $PSI_o = 4.5$ )**

Road j	$PSI_t(j)$	Thin Asphalt <sup>a</sup>			Thick Asphalt <sup>b</sup>		
		$D_{AC}(j)$ (cm)	$h_o(j)$ (cm)	$h_e(j)$ (cm)	$D_{AC}(j)$ (cm)	$h_o(j)$ (cm)	$h_e(j)$ (cm)
1	3.26	8	2.5	2.64	14	4.5	4.90
2	3.03	8	3.0	3.10	14	5.0	5.41
3	2.72	8	4.0	3.71	14	6.0	5.93
4	2.89	7	3.5	2.96	13	5.5	5.26
5	2.53	7	4.0	3.57	13	6.0	5.75
6	2.95	8	3.5	3.26	14	5.5	5.56
7	2.58	8	4.0	3.99	14	6.0	6.13
8	2.27	8	4.5	4.58	14	6.5	6.46
9	1.98	9	5.5	5.75	15	7.5	7.15
10	2.19	8	5.0	4.72	14	7.0	6.53
11	2.06	9	5.0	5.58	15	7.0	7.10
12	1.95	9	5.5	5.81	15	7.5	7.17

<sup>a</sup> Optimal K = 1.24, minimal SSE = 1.22

<sup>b</sup> Optimal K = 3.74, minimal SSE = 0.94

**Table 8. Sample resurfacing thickness using international roughness index (IRI) as a key performance indicator ( $IRI_o = 1.0$  m/km)**

Road j	$IRI_t(j)$ (m/km)	Thin Asphalt <sup>a</sup>			Thick Asphalt <sup>b</sup>		
		$D_{AC}(j)$ (cm)	$h_o(j)$ (cm)	$h_e(j)$ (cm)	$D_{AC}(j)$ (cm)	$h_o(j)$ (cm)	$h_e(j)$ (cm)
1	1.782	8	2.5	2.69	14	4.5	4.90
2	2.087	8	3.0	3.26	14	5.0	5.48
3	2.537	8	4.0	3.87	14	6.0	5.99
4	2.284	7	3.5	3.11	13	5.5	5.33
5	2.838	7	4.0	3.66	13	6.0	5.76
6	2.198	8	3.5	3.43	14	5.5	5.64
7	2.757	8	4.0	4.11	14	6.0	6.15
8	3.290	8	4.5	4.57	14	6.5	6.41
9	3.860	9	5.5	5.55	15	7.5	7.05
10	3.440	8	5.0	4.67	14	7.0	6.46
11	3.695	9	5.0	5.44	15	7.0	7.01
12	3.923	9	5.5	5.59	15	7.5	7.06

<sup>a</sup> Optimal K = 0.71, minimal SSE = 0.72

<sup>b</sup> Optimal K = 2.08, minimal SSE = 1.21

(6 cm) to the original asphaltic surface thicknesses. The observed resurfacing thicknesses,  $h_o(j)$ , for thick asphaltic surface are achieved by adding (2 cm) to the corresponding values associated with thin asphaltic surface. Table 7 provides the results of the minimization of SSE assuming 4.5 initial PSI value. The derived optimal (K) values are (1.24 & 3.74) with (1.22 & 0.94) corresponding minimal SSE considering thin and thick asphaltic surfaces, respectively. Table 8 presents the results of minimizing SSE assuming (1.0 m/km) initial IRI value. The reached optimal (K) values are (0.71 & 2.08) with the corresponding minimal SSE of (0.72 & 1.21) for thin and thick asphaltic surfaces, respectively. Similar results are provided in Table 9 using 100 initial PCR value with (1.24 & 3.83) optimal (K) values and (1.31 & 1.32) minimal SSE considering thin and thick asphaltic surfaces,

**Table 9. Sample resurfacing thickness using pavement condition index (PCI) as a key performance indicator ( $PCI_o = 100$ )**

Road j	$PCI_t(j)$ (cm)	Thin Asphalt <sup>a</sup>			Thick Asphalt <sup>b</sup>		
		$D_{AC}(j)$ (cm)	$h_o(j)$ (cm)	$h_e(j)$ (cm)	$D_{AC}(j)$ (cm)	$h_o(j)$ (cm)	$h_e(j)$ (cm)
1	73.6	8	2.5	2.53	14	4.5	4.84
2	67.9	8	3.0	3.05	14	5.0	5.41
3	59.2	8	4.0	3.82	14	6.0	6.06
4	61.2	7	3.5	3.19	13	5.5	5.51
5	53.4	7	4.5	3.78	13	6.5	5.91
6	62.3	8	3.5	3.55	14	5.5	5.86
7	54.2	8	4.5	4.26	14	6.5	6.33
8	48.0	8	5.0	4.78	14	7.0	6.58
9	41.9	9	5.5	5.94	15	7.5	7.23
10	46.3	8	5.0	4.92	14	7.0	6.33
11	43.2	9	5.5	5.82	15	7.5	7.20
12	41.1	9	5.5	6.01	15	7.5	7.25

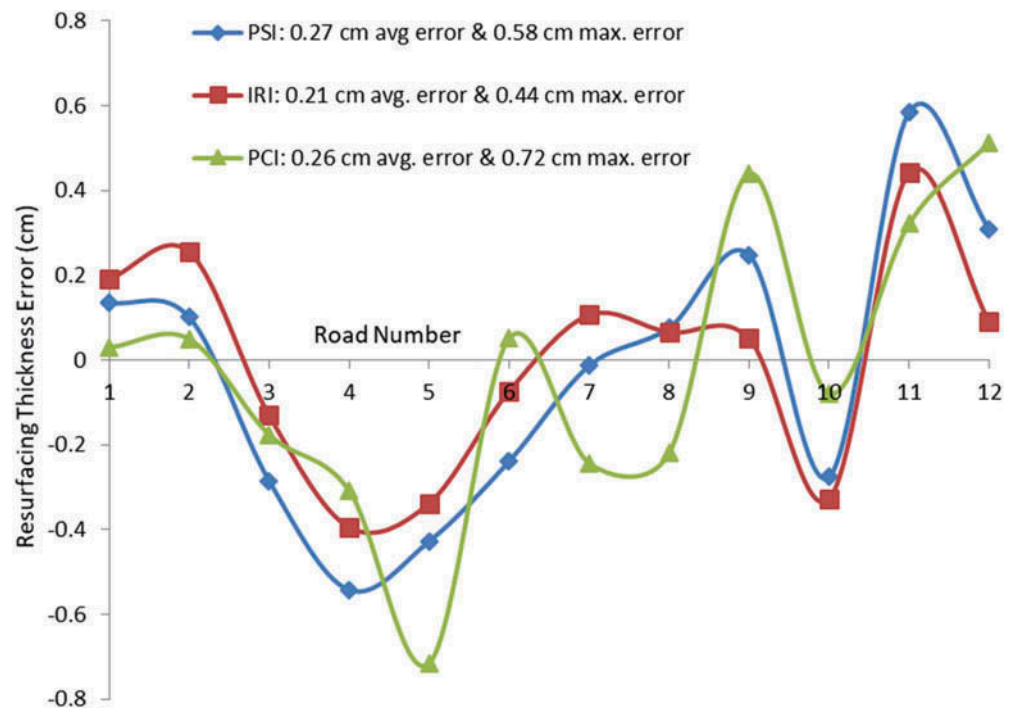
<sup>a</sup> Optimal K = 1.24, minimal SSE = 1.31

<sup>b</sup> Optimal K = 3.83, minimal SSE = 1.32

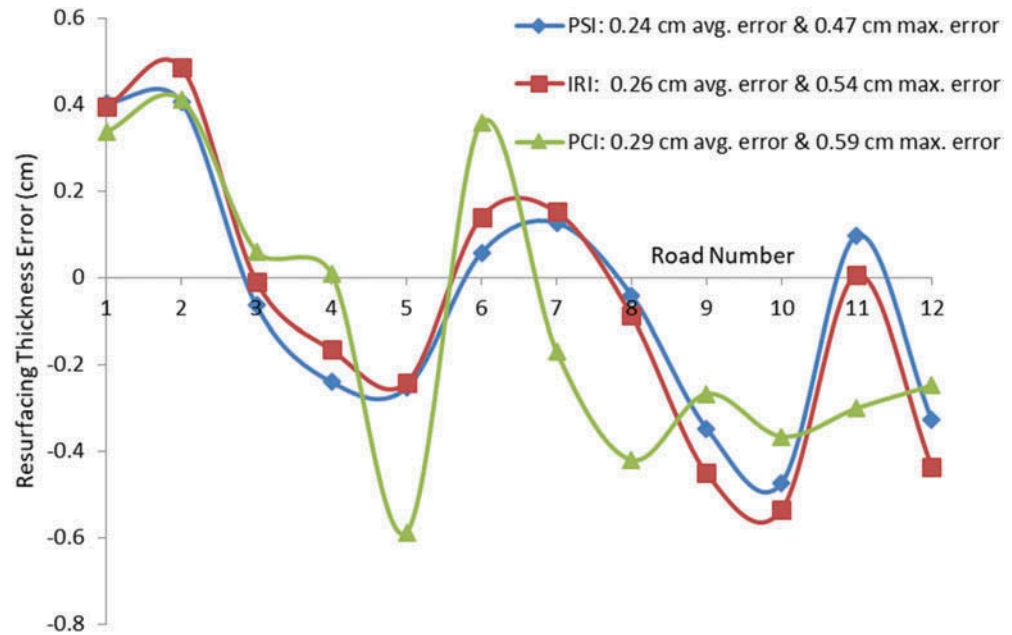
respectively. The results indicate that the optimal (K) values of (1.24, 0.71 & 1.24) associated with thin asphaltic surface are significantly lower than the corresponding (K) values of (3.74, 2.08 & 3.83) for thick asphaltic surface when using PSI, IRI and PCI as KPIs, respectively.

Tables 7–9 also provide the estimated resurfacing thicknesses,  $h_e(j)$ , determined using the derived optimal (K) values, which are very close in values to the corresponding observed resurfacing thicknesses,  $h_o(j)$ . Figures 5 and 6 show the resurfacing thickness errors for the investigated roadway sample with the error being defined as the difference between the estimated resurfacing thickness,  $h_e$

**Figure 5. Sample resurfacing thickness errors estimated for thin asphaltic surface using key performance indicators.**



**Figure 6. Sample resurfacing thickness errors estimated for thick asphaltic surface using key performance indicators.**



(j), and observed resurfacing thickness,  $h_o(j)$ . Figure 5 shows the errors associated thin asphaltic surface to be reasonably low with average values of (0.27, 0.21 & 0.26 cm) and absolute maximum values of (0.58, 0.44 & 0.72 cm) utilizing the PSI, IRI, and PCI as KPIs, respectively. Similarly, Figure 6 shows the average and maximum resurfacing errors to be (0.24, 0.26 & 0.29 cm) and (0.47, 0.54 & 0.59 cm) for thick asphaltic surface using the PSI, IRI, and PCI as KPIs, respectively. Therefore, the resurfacing thickness errors are very much similar in values for both thin and thick asphaltic surfaces.

In addition, the sample resurfacing thickness errors presented in Figures 5 and 6 indicate a high compatibility among the three deployed KPIs in estimating the resurfacing thickness for pavement rehabilitation applications considering a sample of local roadways. However, the curve trends presented in Figures 5 and 6 show a close agreement between PSI and IRI compared to PCI. This indicates that the deployed PSI and IRI values are highly compatible to each other, which may not necessarily be the case with PCI.

#### 4. Conclusions and recommendations

The two case studies presented in the previous section have indicated the effectiveness of the proposed simplified empirical approach in predicting the asphaltic remaining strength factor. The predicted remaining strength factor has been successfully used in estimating the resurfacing thickness component due to strength loss considering both thin and thick asphaltic surfaces. The sample results have indicated that the remaining strength factor is lower for thin asphaltic surface compared to a thick one with the difference becomes larger as the rehabilitation time ( $n$ ) increases, and the factor is lower for inferior pavement performance compared to a superior one as would be expected. The calibration results for a local roadway sample have indicated that the three deployed key performance indicators (PSI, IRI & PCI) resulted in similar resurfacing thicknesses, thus indicating their compatibility. The calibration results have also indicated that the optimal ( $K$ ) values for thin asphaltic surface (0.71–1.24) are considerably lower than the corresponding optimal ( $K$ ) values for thick surface (2.08–3.83). While the paper proposes distinct models for thin and thick surfaces, it is recommended that when unable to classify surfaces as thin or thick (i.e. border cases) to use the solution that gives the highest resurfacing thickness.

The rehabilitation scheduling time ( $n$ ) must be less than the design period ( $N$ ) as shown in Figure 1. The sample presentation has applied practical rehabilitation scheduling times in the range of 6–12 years for

demonstration purposes; however, a life-cycle cost analysis (LCCA) is typically performed to yield the optimal rehabilitation time to be used in estimating the relevant remaining strength factor. Occasionally, some highway agencies select the rehabilitation scheduling time that corresponds to a terminal PSI or IRI value. The remaining strength factor can be estimated for both thin and thick asphaltic surfaces as a different model is proposed in each case. The remaining strength factor mainly represents the proportion of strength remaining in the asphaltic surface, which is typically higher in the case of thick surface. This is because the model for thick asphaltic surface recognizes that pavement distress severity decreases with the increase in depth as evidenced from the surface conditions before and after cold milling. Therefore, the remaining strength factor is generally higher for thick surfaces compared to thin ones as supported by the sample results. Of course, there is a limitation on the cold milling thickness in relation to the thickness of existing asphaltic surface. In practice, cold milling thickness rarely exceeds 5 cm and should not exceed half the existing surface thickness, but also the remaining asphaltic thickness should not decrease below the minimum effective thickness of typically 4–5 cm.

The presented simplified empirical approach can easily be calibrated against potential key performance indicators such as PSI, IRI, and PCI, which makes it useful to most highway agencies. In particular, it is highly beneficial to local governments that mostly rely on collecting periodical pavement distress data using simple field measurements. The outcome of the pavement distress assessment can be converted into a reliable key performance indicator such as the PCI. In addition, the proposed empirical model requires basic design data about the existing pavement structure that is readily available to highway agencies. Therefore, the data requirement for using the proposed empirical approach is minimal and the calculations involved are also very simple to perform. The resurfacing thickness component due to traffic growth can be estimated by mainly relying on empirical design methods such as AASHTO and Caltrans; however, the mechanistic-empirical approach can as well be used. The presented sample results have indicated that this thickness component is relatively small compared to the thickness component due to strength loss especially in the case of inferior performance.

The calibration model associated with the remaining strength factor involves applying the minimization of the sum of squared errors (SSE) to yield the model exponent (K). The minimization procedure of SSE is typically applied to a pavement network (i.e. pavement group) with similar traffic loading conditions and pavement structure characteristics. Therefore, it is recommended that a unique model exponent (K) be derived for each pavement group considering both thin and thick asphaltic surfaces. The asphaltic surfaces associated with low volume roads are generally considered as thin surfaces whereas the asphaltic surfaces for high volume roads can typically be classified as thick surfaces. The main data requirement for minimizing the SSE are the observed resurfacing thicknesses for a roadway sample with reasonable size, which can be obtained from any reliable overlay design method including the “prescription method” typically used by local governments. A minimum roadway sample size of (10–15) projects has proven to be adequate in yielding reliable optimal (K) values with reasonably low resurfacing thickness errors as demonstrated in the investigated local roadway sample. The execution of the minimization of SSE as outlined in this paper is a straightforward procedure and can easily be programmed using commercially available software packages such as “Excel”.

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